Performance-based Service Life Design of Reinforced Concrete Structures Exposed to Chloride Environments

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ABSTRACT: Design for durability is a critical task for reinforced concrete structures exposed to chloride bearing environments. Besides the selection of the concrete composition and the cover thickness, often also preventative measures, such as the use of corrosion resistant rebars or cathodic prevention, are selected. The evaluation of the convenience of any type of preventative measure should be based on a life cycle cost analysis. Nevertheless, the preliminary step for the evaluation of different protection strategies should be a quantitative analysis of their performance in terms of probability of reaching the design service life. Probabilistic performance-based models for service life design are now available, which could also allow the implementation of preventative measures. The paper describes the possible advantages and limitations of such models in the study of different options able to fulfil requested durability requirements.

1 INTRODUCTION

Since the beginning of the last Century, the combined use of reinforced concrete became a worldwide common practice in the construction of structures and infrastructures. Unfortunately, the widespread use of reinforced concrete has raised serious durability problems as a consequence of corrosion of steel reinforcement (Tuutti 1982, Page and Treadaway 1982, Arup 1983, Schiessl 1988, Bertolini et al. 2004). Nowadays, durability has become a critical issue in the management of new and existing reinforced concrete structures. In the case of new constructions, prevention of steel corrosion has to be taken into consideration since the design stage. For this reason, tools aimed at the design of durable structures as well as quality control procedures at the construction site, which are essential for obtaining a durable structure, have been developed.

This paper summarizes the main aspects of corrosion of steel reinforcing bars in concrete and describes possible approaches for the design of durable structures. Performance-based approaches are discussed in order to show their usefulness in the determination of different design options which allow to fulfill durability requirements. The Model code for service life design proposed by the International federation of structural concrete (FIB) will be considered to show some examples of design of structural elements exposed to a marine environment, taking into account concrete composition, cover thickness and additional protections such as galvanized or stainless steel bars.

2 CORROSION OF REBARS AND ITS EFFECTS

Steel in sound concrete is protected by the alkaline solution contained in the pores of the hydrated cement paste, which promotes passivation, i.e. the formation of a spontaneous thin protective oxide film on the surface of the steel. Under this condition, corrosion rate is negligible even if the concrete is permeated by oxygen and moisture.

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(*a*)

(*b*)

Figure 1 – Examples of the effects of corrosion of steel in concrete: *(a)* spalling of the concrete cover due to carbonation induced corrosion, *(b)* localised attack on a bar due to pitting corrosion in chloride contaminated concrete.

Corrosion can, however, take place when the passive film is removed or is locally damaged. In chloride environments this usually occurs due to chloride penetration; this phenomenon will be described further on. Other forms of corrosion, e.g. due to carbonation, stray currents or hydrogen embrittlement (Bertolini et Al. 2004), will not be considered in this paper. Corrosion may have several consequences on the serviceability and safety of reinforced concrete structures. Oxides produced at the steel surface can produce tensile stresses in the concrete cover, which may lead to cracking, spalling in localized areas, or delamination (Figure 1*a*). Reduction of bond of the reinforcement to the concrete may also occur. In the case of localized corrosion, the cross section of the reinforcement can be significantly reduced (Figure 1*b*) and thus the load-bearing capacity of a structural element, its ductility and seismic behaviour as well as its fatigue strength may be affected, even before any cracking takes place in the concrete cover.

2.1 Chloride ions

When chloride ions, which are contained for instance in seawater or in common de-icing salts, penetrate the concrete cover and reach a critical value (chloride threshold level) at the depth of the reinforcement, a localized attack can take place (named pitting corrosion, Figure 1*b*).

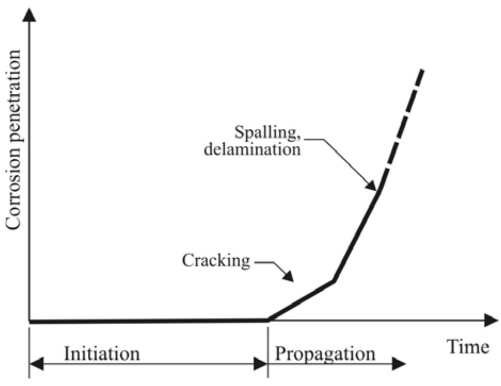

Figure 2 – Example of evolution in time of corrosion-related effects in reinforced concrete structures (adaptation of Tuutti’s diagram).

Figure 2 depicts the effects of corrosion on the life of a reinforced concrete structural element. In a first stage the steel reinforcement is passive and no corrosion takes place. Chlorides (or carbonation), however, beginning from the concrete surface, penetrates the concrete cover. Corrosion initiates when the critical threshold level is reached at the

surface of the steel reinforcement. Even though it does not in itself affect the serviceability or the stability of the structure, corrosion initiation is a critical time in the life of the structure. In fact, the depassivated steel becomes susceptible to corrosion with a rate that depends on environmental factors. In time, corrosion products will cause reduction in the cross section of the bars or cracking, spalling and delamination of the concrete cover, which may compromise the serviceability and the stability of the structure.
As far as corrosion of steel is concerned, a service life can be defined as the sum of the initiation time and the propagation time (Tuutti, 1982). The *initiation period* can be defined as the time required for chlorides to penetrate and depassivate the steel. The *propagation period* begins when the steel is depassivated and finishes when a given limit state is reached beyond which consequences of corrosion cannot be further tolerated. This distinction between initiation and penetration periods is useful in the design of reinforced concrete elements, since different processes and variables should be considered in modeling the two phases.
The risk of chloride-induced corrosion is usually associated with the penetration of chlorides through the concrete cover. In simple words, pitting corrosion initiates when the penetration of chlorides is such that the threshold value is reached at the steel surface, namely at a depth equal to the cover thickness. This is depicted schematically in Figure 3. In practice, however, evaluation of the initiation time is quite a complicated task, because of a large number of variables that influence both the kinetics of chloride penetration through the concrete cover and the chloride threshold value.

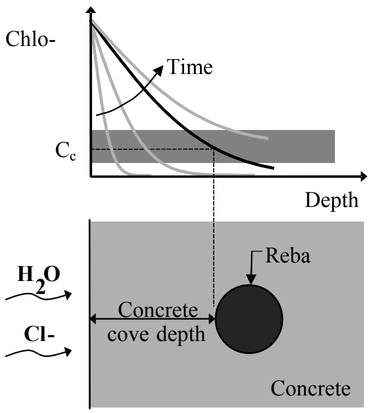

Figure 3 – Initiation of chloride induced corrosion and factors involved.

Chloride penetration from the environment produces a profile in the concrete characterized by high chloride content near the external surface and decreasing contents at greater depths. The profile that can be obtained in time in a specific point of a given RC element depends on many factors, the main ones being related to: the concrete properties, the mechanisms of transport of chloride bearing solutions, the moisture content of concrete, and the concentration of chlorides in the environment. Depending on local exposure conditions, transport of chlorides through the concrete cover may take place due to diffusion, capillary suction, permeation, and possibly migration (CEB 1992, Frederiksen 1996, Bertolini et al. 2004). In real structures, the transport of chlorides through concrete often takes place by a combination of these transport mechanisms. For example, when a structural element is exposed to wetting-drying cycles, it is subjected to capillary absorption of the chloride-bearing solution during wetting, possibly followed by diffusion during the wet period, while during dry periods evaporation of water brings about accumulation of chlorides near the surface.
Exposure to precipitations, conversely, may wash out chlorides in the surface of concrete. Chloride penetration in a reinforced concrete structure is thus a complex

function of geometry, position, environment and concrete composition. The complex nature of transport of chlorides in concrete and the difficulty in evaluating appropriate values of the relevant transport parameters has lead to the adoption of simplified procedures. The experience on both marine structures and road structures exposed to de-icing salts, has shown that, in general, chloride profiles can be reasonably described by means of the following relationship:

$$C(x,t) = C_s\left[1-\mathrm{erf}\left(\frac{x}{2\sqrt{Dt}}\right)\right] \tag{1}$$

where: $C(x,t)$ is the chloride concentration at depth x and time t, C_s is the surface concentration of chlorides and D_{app} is an apparent diffusion coefficient. This is a solution of Fick's second law of diffusion under the assumptions that concrete does not initially contain chlorides, that the concentration of the diffusing chloride ions, measured on the surface of the concrete, is constant in time and is equal to C_s, that the coefficient of diffusion D is constant in time and does not vary through the thickness of the concrete. This relationship was firstly proposed by Collepardi et al. (1972) to fit profiles of penetration of chlorides in concrete under diffusion conditions. As a matter of fact, only in concrete completely and permanently saturated with water, chloride ions can penetrate by pure diffusion. In most situations, as it was previously described, other transport mechanisms contribute to chloride penetration (e.g. capillary suction) while binding with constituents of the cement paste may alter the concentration of free chlorides in the pore solution. In spite of this, the apparent diffusion coefficient, obtained from real structures or laboratory tests, is often also used as a parameter to compare the resistance to chloride penetration of different concretes, assuming that the lower D_{app} the higher the resistance to chloride penetration. It should, however, be observed that this parameter does not only depend on concrete properties, but it is also influenced by exposure conditions or the time of exposure.

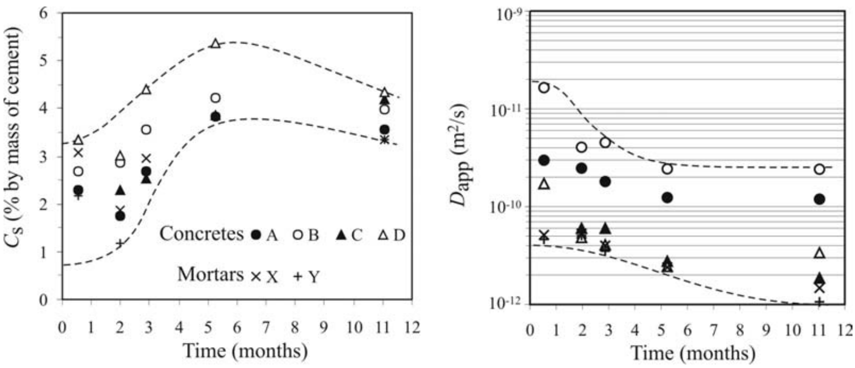

Figure 4 – Example of changes of C_s and D_{app} calculated by fitting chloride profiles measured on concrete specimens measured after different times of exposure to wetting/drying cycles with 3.5% NaCl solution. *A*: portland cement, $w/c = 0.5$, *B*: portland cement, $w/c = 0.65$, *C*: slag cement, $w/c = 0.5$, *D*: slag cement, $w/c = 0.65$, *X*: pozzolanic cement, $w/c = 0.4$ (repair mortar); *Y*: proprietary repair mortar (Bertolini et al., 2002).

Therefore, results obtained under particular conditions, especially during short-term laboratory tests, may not be applicable to other environments or to longer periods of exposure. For instance Figure 4 shows values of C_s and D_{app} measured by fitting chloride profiles in concretes and mortars of different composition, after different times of exposure to simulated marine tidal zone (specimens were subjected to alternate

wetting with 3.5% NaCl solution and drying at 40°C; for details see Bertolini et al., 2002).
Changes of about one order of magnitude in the apparent diffusion coefficient can be observed between chloride profiles measured after 1 month of exposure and after one year of exposure. Therefore, even if differences between the various materials are evident, e.g. higher diffusion coefficient was observed for materials with higher *w/c* ratio and portland cement (see caption of the figure for details), the actual value of D_{app} of each material remarkably changes in time.
Relationship (1) has also been proposed for the prediction of long-term performance of structures exposed to chloride environments. With this regards it should be stressed again that D_{app} and C_s, in general, cannot be assumed as constants in the case of real structures where binding as well as processes other than diffusion take place. Corrective factors are normally proposed in service life models (FIB, 2006).
As far as the chloride threshold values (Cl_{th}) is concerned, it should be pointed out that it depends on numerous factors (Glass and Buenfeld, 1997, Angst et al. 2009). Major factors have been identified in the potential of the steel, the pH of pore solution in the concrete, and the presence of microstructural defects at the steel/concrete interface. The influence of the potential of steel on the chloride threshold is depicted by the Pedeferri diagram shown in Figure 5. The electrochemical potential of steel is primarily related to the moisture content of concrete, which determines the availability of oxygen at the steel surface. In structures exposed to the atmosphere (case 1), oxygen can easily reach the steel surface through the air filled pores and the corrosion potential of the reinforcement is around –200/0 mV vs SCE (saturated calomel reference electrode).

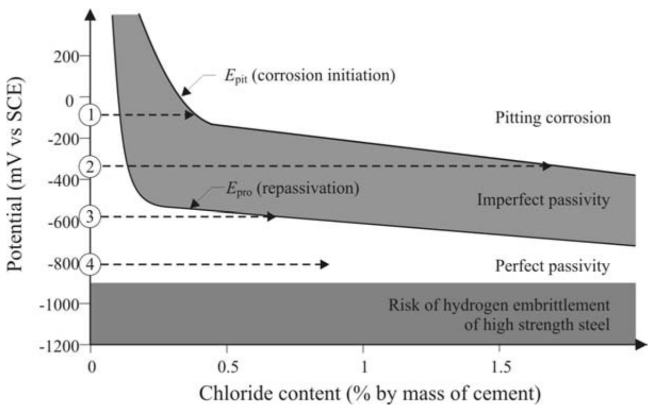

Figure 5 – Pedeferri Diagram, showing the effect of steel potential on the chloride threshold level (adapted from Pedeferri, 1995). Different electrochemical conditions of steel are depicted: 1 = steel in concrete exposed to the atmosphere, 2 = steel subjected to cathodic prevention, 3 = steel subjected to cathodic protection, 4 = steel in submerged concrete.

Since first investigations on real structures were carried out (Vassie, 1984), it was shown that the risk of corrosion in non-carbonated concrete may be considered low for chloride contents below 0.4% by mass of cement (total chloride content). When a reinforced concrete element is saturated by water, the transport of oxygen to the steel is low in the pores filled by water and the reinforcement reaches very negative potentials (e.g. lower than –600/–800 mV vs SCE, case 4). Under this circumstance, the chloride threshold is greater than in aerated structures, sometimes even reaching values one order of magnitude higher. For this reason, parts of RC structures permanently immersed in seawater rarely experience pitting corrosion initiation. A lowering in the steel potential, and consequently an increase in the chloride threshold value, can also be induced by an external current that polarizes cathodically the steel (Pedeferri, 1995),

such as in the case of the application of cathodic prevention (case 2) or cathodic protection (case 3).
Furthermore, the chloride threshold is a function of the pH of the pore solution, which depends on the type of cement and additions, although the effect of these parameters is not clear from the literature. The chloride threshold has also been found to be dependent on the presence of defects in the concrete near the steel surface (such as macroscopic voids or interfacial transition zone). However, it should be observed that, since initiation of pitting corrosion is known to be a statistical process, the chloride threshold can only be defined on a statistical basis (Cost 521, 2003).
A technical committee of Rilem is dealing with the difficult task of defining a standard procedure for the evaluation of the chloride threshold (www.rilem.org, TC 235-CTC).
Once pitting corrosion has initiated, it may propagate very fast, leading to a penetrating localised attack (Figure 1*b*). The propagation of the attack may be further accelerated by macrocell that forms between areas with different electrochemical behaviour (e.g. in hollow marine structures with air inside, corrosion may be stimulated by a macrocell on the bars in the outer parts by passive bars embedded in aerated concrete in the inside).

3 CORROSION PREVENTION

Different service life strategies have been developed in recent years for reinforced concrete structure in order to meet target service life in the design of new structures or prolong the service life of existing ones (Bertolini, 2008). These will be briefly described.
According to recent design codes, a durable structure shall meet given requirements of serviceability, strength and stability throughout its service life, without significant loss of utility or excessive unforeseen maintenance. Therefore, long-term effects of corrosion of steel bars should also be taken into account in the design stage, in order to avoid that any relevant damage will be reached during the required service life, also considering the intended use of the structure, the maintenance program and actions. Basically, this requires that a suitable limit state related to steel corrosion has to be selected, in order to define the end of the service life. Cracking or detachment of the concrete cover is usually considered in the case of carbonation-induced corrosion, which produces uniform attack (Figure 2). Initiation of corrosion is often chosen as limit state for chloride-induced corrosion, owing to the localized nature of pitting attack, which, once it has initiated, can bring about rather quickly a marked reduction in the cross-section of the bars even in the absence of any external damage on the concrete cover. Factors that influence the service life of a structure can be summarized in:
- loads applied to the structure (both mechanical actions and environmental actions);
- concrete properties (water/cement ratio, type of cement, cement content, workability, compaction, curing, quality controls at the construction site, cracking),
- thickness of the concrete cover,
- design (structural conception, construction details, etc.),
- additional preventative measures (galvanized or stainless steel bars, surface treatment of concrete, cathodic prevention, etc.)
- planned controls or maintenance (regular inspection, monitoring, replacement of non-structural parts, reapplication of a coating, etc.).

It is not possible here to describe in details all the options available during the design stage. It suffices to say that nowadays both standard approaches based on deemed-to-satisfy rules and performance based approaches are available. For instance, in European standards a standardized method to deal with durability, is proposed which is based on the definition of an exposure class and the subsequent prescriptions regarding the *w*/*c* ratio, the cement content and the thickness of the concrete cover. Tables providing maximum or minimum values for these parameters are provided in EN 206-1 and EN 1992-1-1 standards, which are normally associated to an intended service life of about 50 years.
For structures exposed to aggressive environments, which are mainly related to the presence of chlorides, such deemed-to-satisfy rules would lead to adopting too much

restrictive prescriptions in those parts of the structure that are not under the most aggressive exposure conditions. In this case a tailored design for durability would be much more appropriate. The designer, on the basis of both the general exposure conditions of the structure and the microclimate, should design every structural element in a way that it can withstand the actual local conditions of exposure during the required service life. To do this, modeling of degradation mechanisms due to attack by a particular aggressive agent is required, in order to estimate the evolution of deterioration depending on the influencing factors.

Methods using probabilistic approaches have been proposed, which allow a quantitative evaluation of the service life of a structure with respect to reinforcement corrosion. Some of them, like that proposed by the International Federation for structural concrete (FIB, 2006), are based on a probabilistic or semi-probabilistic approach similar to that used in the structural design: limit states that indicate the boundary between the desired and the adverse behavior of the structure are defined. In these models the concrete behavior, for instance its resistance to chloride penetration, is evaluated by means of an accelerated test, which provides an apparent chloride diffusion coefficient. However the test result cannot be directly used to predict the chloride penetration in a structure, but should be corrected through coefficients that take into account, for instance, the real environmental exposure conditions. Design equations have been set to calculate the failure probability of preset performances of the structure as a function of time. The acceptable probability should be selected on the basis of the severity of the adverse event occurring (limit state), although this task is not simple.

Though the definition of quality and thickness of the concrete cover is the first step in the design of a durable reinforced concrete structure, under strong environmental aggressiveness and/or when a long service life is required (e.g. 100 years or more), the designer can take advantage in the use of additional protections. For instance, in chloride bearing environments the chloride threshold value can be increased by using corrosion resistant steel (e.g. stainless steel, Bertolini and Pedeferri, 2002), by decreasing the steel potential by applying the technique of cathodic prevention (Pedeferri, 1995) or using corrosion inhibitors. Although preventative techniques increase the initial cost of the structure, they may lead to a reduction in the overall costs throughout the required service life. Significant reduction in the costs can be obtained by applying the additional protection only to the most critical parts of the structure, while protection of bars in other, less aggressive, zones is provided only by the concrete cover. The life cycle costs analysis is often used for the evaluation of the convenience of preventative techniques.

Beyond economical aspects, the use of additional protections may have the advantage of increasing the reliability of the structure. It has been questioned whether relying entirely on the protective properties of a few centimetres thickness of the concrete cover in severe chloride-laden environments is really the most effective way of ensuring that embedded steel remains free from significant corrosion for very long periods of time (Page, 2002). Taking also into account that a reinforced concrete structure has to be designed to fulfill many other functions than protecting embedded steel, the application of additional protections may be advantageous. The selection of the appropriate preventative technique among those nowadays available should take into account also the reliability and the track record of each technique. It is not possible to treat this aspect in this paper, and reference to specialized literature is made (COST 509 1997, COST 521 2003, Bertolini et al. 2004).

Quality of the execution of concrete is of primary importance in order to achieve the performance requirements assumed in the design of the structure. For instance, the advantages of a lower *w/c* ratio or the use of blended cement can only be achieved if concrete is properly placed, compacted and cured. It should be stressed that poor curing will mainly affect the concrete cover, i.e. the part that is aimed at protecting the reinforcement, since this is the part most susceptible to evaporation of water. Similarly, low quality controls on the thickness of the concrete cover may have dramatic consequences on the time to corrosion initiation.

4 USE OF PERFORMANCE-BASED MODELS

Through the use of a performance-based model different design options can be compared and the one that determines the best compromise between the needs related to different aspects of design (as, for example, structural and economic ones) can be selected. In order to show the utility of performance-based models for service life design, some examples will be considered of the application of the FIB model (FIB 2006) to structures exposed to marine environments (examples are described in more details in Bertolini et al. 2010).

If we consider a structure subject to chloride-induced corrosion, the onset of corrosion can be assumed as the limit state, as for structural elements exposed in the marine environment the propagation of corrosion is short and it can be neglected in the design stage. The state limit is reached when the protective film that covers the rebar is broken, i.e. when the chloride concentration $C(x,t)$ at time t and at the depth of the reinforcement x is equal to the critical chloride threshold C_{cr}. The initiation time can be calculated by solving the probabilistic equation:

$$P_f = P\{C_{cr} - C(x,t) < 0\} < P_o \tag{2}$$

where P_f is the failure probability and P_o is the maximum acceptable probability of failure. In addition to the probability of failure, it may be also calculated, as a measure of reliability, the safety index β, defined as:

$$\beta = -\Phi^{-1}(P_f) \tag{3}$$

where Φ^{-1} is the inverse of the normal distribution.

Figure 6 shows a possible example of the evolution in time of variables C_{cr} and $C(x,t)$. The intersection of the two probability distributions allows to assess the probability distribution of the service life. The intersection of the average values of the two curves defines the other hand, the average service life (t_u^*), i.e. the instant at which the probability of reaching the limit state is 50%. Depending on the acceptable probability of failure, P_o, therefore, the service life (equal to the time of initiation t_i) can be defined.

4.1 Concrete composition and cover thickness

The selection of composition of concrete is the first step in the design of the durability of a reinforced concrete structural element, not only for the prevention of corrosion of reinforcement, but also for the prevention of the deterioration of the concrete (Collepardi, 2006).

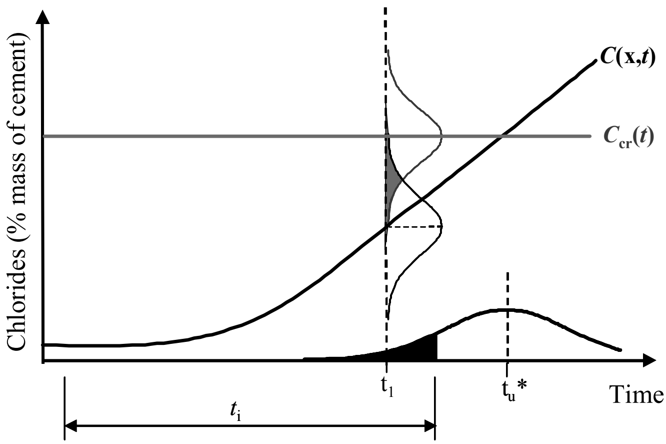

Figure 6 - Example of a probabilistic approach for the evaluation of service life for a structure subject to chloride-induced corrosion.

A design service life of 50 years and a limit state corresponding to corrosion initiation will be considered. For the application of the model, factors that determine the initiation time should be defined and for each of them an appropriate probability distribution should be identified. It is not possible here reporting in detail the equations and parameters of the model (FIB 2006). The factors are, however, summarized in Table 1, which also contains parameters that characterize the probability distributions considered for the cases described in the examples here described (Bertolini et al. 2010).

Table 1 – Input parameters considered in the application of the FIB *Model Code for Service Life Design* for the design of structural elements exposed to the splash zone (Bertolini et al. 2010).

Parameter	*Units*	*Description*	*Distribution**	*Alternative*	*Average value*	*Standard deviation*
C_{cr}	% mass cement	chloride threshold	BetaD $0.2 \leq C_{cr} \leq 2$	carbon steel	0.6	0.15
			BetaD $0.6 \leq C_{cr} \leq 3$	galvanized steel	1.2	0.3
			BetaD $3 \leq C_{cr} \leq 8$	Stainless steel 1.4307	5	0.5
			BetaD $5 \leq C_{cr} \leq 10$	Stainless steel 1.4462	8	0.5
$C_{s,\Delta x}$	% mass cement	chloride content at depth *Δx*	LogND	-	6.0	1.0
C_0	% mass cement	initial chloride content	D	-	0.15	-
d_c	mm	cover depth	ND	-	to be determined	6
Δx	mm	depth of convection zone	BetaD $0 \leq \Delta x \leq 50$	-	8.9	5.6
k_e b_e	K	regression variable	ND	-	4800	700
T_{rel}	K	temperature of element	ND	-	293	10
				-	303	10
T_{ref}	K	standard test temperature	D	-	293	-
$D_{RCM,0}$	10^{-12} m^2/s	chloride migration coefficient	ND	OPC	6.5	1.3
				BF	1.5	0.30
				CA	13	2.6
A(t) *a*	-	ageing factor	BetaD $0 \leq a \leq 1$	OPC, CA	0.3	0.12
				BF	0.45	0.065
t_0	year	reference time	D	-	0.0767	-
t	year	time	D	-	to be determined	-

* ND: normal distribution; D: deterministic; BetaD: beta distribution; LogND: log-normal distribution.

The performance of concrete in terms of resistance to penetration of chlorides, are "measured" through the accelerated test of inclined migration cell, in which the penetration of chlorides is forced by an electric current (NT-Build, 1999). From this test a diffusion coefficient, called D_{RCM} (Rapid Chloride Migration) is obtained for a specific concrete. For example, for portland cement (OPC, corresponding to a type CEM I, according to UNI EN 197-1), binder with 70% blast furnace slag (BF, corresponding to type CEM III/B), and binder with 30% of ground limestone (LI, corresponding to type CEM II/B-L), experimental results summarized in Figure 7 can be considered.

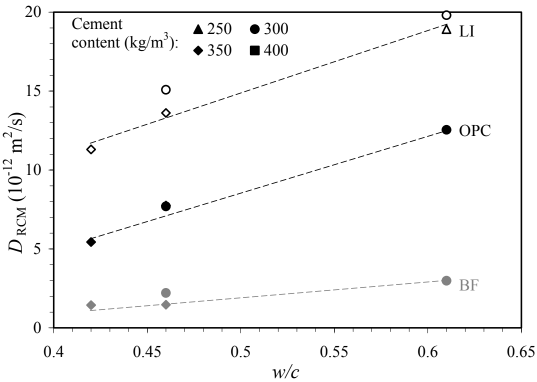

Figure 7 - Chloride diffusion coefficient D_{RCM}, as average of 2 concrete specimens cured 28 days, depending on the water/cement ratio, the type of cement (Portland cement, OPC, cement with 30% limestone, LI, 70% of cement with blast furnace slag, BF) and the dosage of cement (Bertolini et al. 2008).

Once the required service life (50 years) and exposure conditions (such as, for example, considering a temperature of 20°C) are defined, the probability of failure (P_f, corresponding in this case, to the probability of corrosion initiation within 50 years), or the safety index β can be calculated, as a function of the thickness of concrete cover. Figure 8 shows, for example, the results for concrete made with portland, 70% blastfurnace and 30% limestone cements and w/c = 0.45, assuming that, according to Figure 7, they have values of D_{RCM} of $6.5 \cdot 10^{-12}$, $1.5 \cdot 10^{-12}$ and $13 \cdot 10^{-12}$ m^2/s respectively.

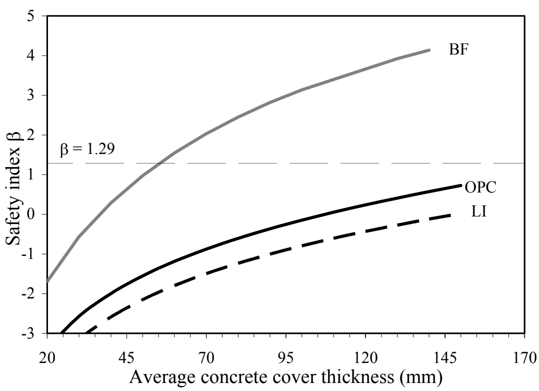

Figure 8 - Thickness of concrete cover required to ensure an average service life of 50 years for a reinforced concrete element exposed in the splash zone, depending on β index and the type of binder (w/c ratio 0.45 and average annual temperature = 20°C) (Bertolini et al. 2010).

To identify possible design solutions, the maximum acceptable probability of failure, P_o, should be defined. FIB model recommends, for the limit state of chloride-induced corrosion initiation, the value of 10^{-1} (P_o = 10%), which corresponds to a safety factor of β = 1.29. Figure 8 shows that, in the example here considered, using a binder with 70% blast furnace slag (BF) combined with *w/c* ratio of 0.45 it would be necessary to design a minimum thickness of concrete cover of about 55 mm. For concrete made with portland cement (OPC) and 30% limestone (CA) even *w/c* ratio of 0.45 and a thickness of 150 mm are not sufficient.

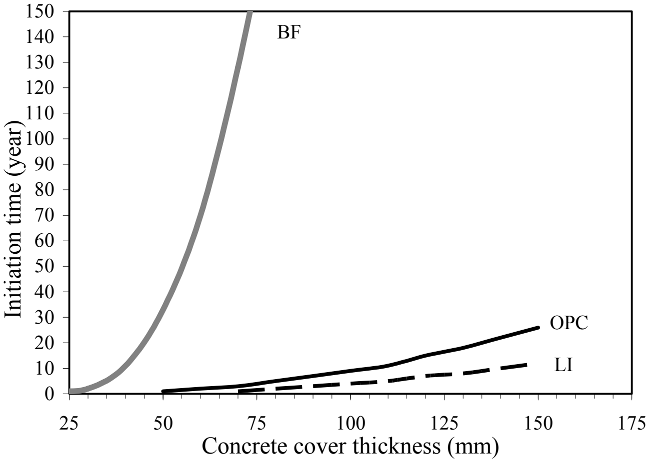

Figure 9 – Time of initiation of corrosion calculated for a safety index of 1.29 as a function of the thickness of concrete cover and the type of binder (*w/c* ratio 0.45, average annual temperature = 20°C) (Bertolini et al. 2010).

Figure 9 shows a different way of using a probabilistic model, applied to the previous example. In this case, the maximum acceptable probability of failure has been fixed at $P_o = 10^{-1}$ (i.e β = 1.29) and the service life (assumed equal to the time that probability of initiation of corrosion is equal to P_o) has been plotted as a function of the thickness of concrete cover. It can be observed that, concrete made with portland cement and 30% ground limestone and *w/c* ratio of 0.45, even with extremely high cover thickness exceeding 100 mm, bring about a service life lower than 10 and 30 years respectively. Much longer service lives could instead be obtained by using 70% blast furnace slag cement and the same *w/c* ratio; for example with a thickness of concrete cover of 75 mm a service life of more than 150 years is estimated.

4.2 Preventative techniques

The use of preventative techniques (Bertolini et al. 2004), such as corrosion resistant rebars, cathodic prevention, corrosion inhibitors, surface treatments of concrete, etc. may be an effective strategy to reduce life cycle costs and increase durability of concrete structures exposed to aggressive environments or for which a service life longer than 50 years is required. Despite a higher initial investment, preventative techniques can save significant costs over the life cycle of the structure. In addition, the improved corrosion resistance can significantly help to reduce the risks associated with premature corrosion failures, often resulting from incorrect placement or curing of concrete, allowing a more reliable prediction of service life design and, consequently, increasing the durability of the structure.
However, while the higher costs associated with the use of preventative techniques can be easily calculated in the design phase, the economic benefits, derived either from savings in future maintenance costs and increasing the reliability of the design solution,

are difficult to determine. The indispensable first step to quantitatively assess the benefits of using these techniques is the assessment of their contribution in determining the actual service life of a structure. Performance based approaches are therefore needed to compare different design solutions.

As an example, we can consider corrosion resistant bars made of stainless steel or galvanized steel. In the case of chloride induced corrosion, the advantage of corrosion resistant rebars is related to the increase in the chloride threshold.

As proposed by the FIB model for ordinary reinforcement, the critical content of chloride can be described by a beta probability distribution (this distribution is characterized by a mean value, standard deviation and a specific range of variation). Although values for the chloride threshold of steel other than carbon steel are not provided by the model, estimation was made for the following types of steel: galvanized steel, stainless steel types 1.4307 (austenitic stainless steel with 18% chromium and 8-10% nickel). Based on data available in the literature and laboratory data the parameters of the distribution shown in Table 1 were defined (Bertolini et al. 2010). Figure 10 shows the results of the model for a reinforced concrete element made with blast furnace slag cement having a diffusion coefficient D_{RCM} of $1.5 \cdot 10^{-12}$ m^2/s, for which a service life of 50 years is required in an environment with an average temperature of 30°C (then a more aggressive environment than that seen in Figures 8 and 9). The figure plots the safety index as a function of the thickness of concrete cover, allowing the determination of combinations of steel type and thickness of concrete cover able to fulfil specific durability requirements. For example, considering a safety index of 1.29 ($P_o = 10^{-1}$), the minimum thickness of concrete cover is reduced from 70 mm for ordinary carbon steel, to 55 mm for galvanized steel, to less than 25 mm for the stainless steel.

Alternatively, a concrete cover thickness of 50 mm (which is a reasonable value for structures exposed to chloride environments) can be considered, and it can be observed that carbon reinforcement is not adequate to prevent corrosion, as β index is around 0, which corresponds to a probability of initiation of corrosion higher than 50%, while the stainless steel bars allow to guarantee, with a confidence level greater than required, the service life of 50 years. This approach allows to quantify the increase in reliability brought about by the use of the corrosion resistant bars.

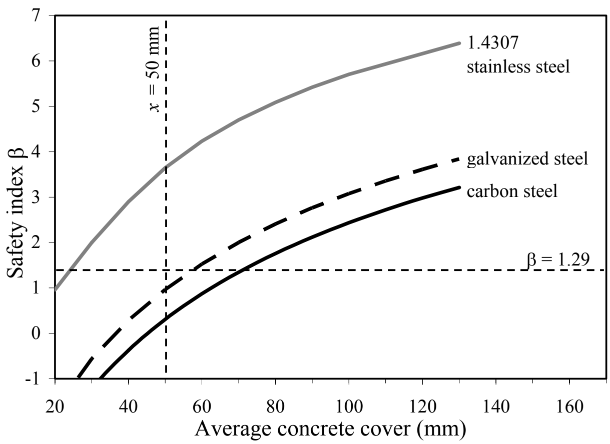

Figure 10 - Thickness of concrete cover required to ensure an average service life of 50 years for a reinforced concrete element exposed in the splash zone, depending on the index β and the type of reinforcement: carbon steel, galvanized steel, and type 1.4307 stainless steel (average annual temperature = 30°C, blast furnace slag cement with *w/c* ratio equal to 0.45) (Bertolini et al. 2010).

5 CONCLUDING REMARKS

Possible use of nowadays available probabilistic models for the design of the durability of reinforced concrete structural elements subject to corrosion by chlorides has been described. This approach, provided for instance by the FIB "Model Code for the service life design" used in the examples described in this paper, allows, in addition to a design of concrete composition to be made in line with structural design, the formulation of durability related prescriptions (e.g., a minimum thickness of concrete cover, a maximum value of D_{RCM} be fulfilled for the quality control) and to take advantage of new materials and additional protections.
This type of approach has, therefore, clear advantages in the evaluation of the life cycle cost of reinforced concrete. Obviously the reliability of the prediction made with the probabilistic models is crucial. In this regard, there are still open aspects to be considered in the future (Gulikers, 2007). The parameters of the models require calibration of real structures, in particular, some factors that modify the result of accelerated tests, have a remarkable influence on the evaluation of service life and seem to overestimate the performances of some types of binder in relation to chloride penetration. In addition, many aspects of the construction phase, such as curing, are not always adequately considered in the models as well as unexpected early cracking of concrete. Another crucial aspect concerns the definition of the probability of failure P_o, since the choice of this parameter has a fundamental role in the output of the mode; possibly the definition of P_o could be related to life cycle cost analysis in order to define optimum values.
Further research is needed in order to collect more experimental data on traditional and new materials to introduce in the models, for instance regarding the chloride threshold levels as a function of concrete and steel composition. Reliability of the output of the models should also be investigated. With this regard, the application of the model to existing structures and the comparison of the results with the present condition of the structure may provide useful information for the validation of the models (Bertolini et al. 2011).

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